DESIGN CRITERIA
FOR
PRESTRESSED CONCRETE ROAD BRIDGES
(POST-TENSIONED CONCRETE)

(*Second Revision*)

THE INDIAN ROADS CONGRESS
1990

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Published by

THE INDIAN ROADS CONGRESS
Jamnagar House, Shahjahan Road,
New Delhi-110 011

1990

Price Rs 20
(Plus packing and postage)

First published : December, 1965
Reprinted : October, 1969
Reprinted : February, 1974
First Revision : January, 1977
Reprinted : July, 1983 (with Notations and units as per IRC : 71)
Second Revision : November, 1985
Reprinted : September, 1990

Printed at Sagar Printers & Publishers, New Delhi
2000 — Sept. 1990

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NOTATIONS

A_s	:	Area of High Tensile Steel
A	:	Area of longitudinal reinforcement
A_{sv}	:	Cross-sectional area of two legs of a link
A_t	:	Area of connector steel
A_0	:	Area enclosed by the centre line of members forming a box section
A_1	:	Bearing area of the anchorage converted in shape to a square of equivalent area
A_2	:	Maxium area of the square that can be contained within the member without overlapping the corresponding area of the adjacent anchorages, and concentric with the bearing area 'A_1'.
b	:	Width of a rectangular section or rib of a Tee, L or I beam
b_1	:	Side of anchor plate
B_f	:	Width of flange of Tee or L beam
d	:	Overall depth of the girder measured from top of deck slab to the soffit of girder
d_b	:	Depth of the girder from the maximum compression edge to the centre of gravity of the tendons
ds	:	Diameter of prestressing wire/strand
dt	:	Depth from extreme compression fibre either to the longitudinal bars or the centroid of the tendons, whichever is greater
E_c	:	Modulus of Elasticity of concrete at 28 days
E_s	:	Modulus of Elasticity of prestressing steel
Ecj	:	Modulus of Elasticity of Concrete at j days ($j<28$ days)
e	:	Base of Naperian Logarithms
F_{bst}	:	Bursting tensile force in end block
f_{cav}	:	Average compressive stress in flexural compressive zone
f_{cj}	:	Actual concrete cube strength at j days subject to a maximum value of fck ($j<28$ days)
f_{ck}	:	Characteristic compressive strength of 15 cm cubes at 28 days
f_{cp}	:	Compressive stress at centroidal axis due to prestress taken as positive
f_b	:	Permissible compressive contact stress in concrete including any prevailing stress as in the case of intermediate anchorages
f_p	:	Ultimate tensile strength of prestressing steel
f_{pt}	:	Stress due to prestress only at the tensile fibre distance 'y' from the centroid of the concrete section
ft	:	Maximum principal tensile stress in concrete
f_{yc}	:	Yield stress of longitudinal steel in compression
f_{yl}	:	Yield strength of longitudinal reinforcement or 0.2 per cent proof stress which should be taken as not greater than 415 MPa
f_{yv}	:	Yield strength of linbs/shear reinforcement or 0.2 per cent proof stress which should be taken as not greater than 415 MPa
G	:	Dead load
H_{max}	:	Larger dimension of the section

H_{min}	:	Smaller dimension of the section
h_{wo}	:	Wall thickness of members of box section where torsional stress is determined
I	:	Second moment of area of the section
k	:	Wobble coefficient per metre length of prestressing steel
l	:	Length of specimen
M	:	Bending moment at the section
MPa	:	Mega Pascals
Mt	:	Cracking moment at the concrete section considered
M_{ult}	:	Moment of section under ultimate load condition
n	:	Number of test strength results
P_k	:	Load in tendon
Q	:	Design live load including impact
S	:	Standard deviation (Sample)
SG	:	Superimposed dead load
S_t	:	Spacing of connectors
S_v	:	Link spacing along the length of a member
T	:	Torsional moment due to ultimate loads
t	:	Thickness of flange of a Tee beam
V	:	Shear force at the section considered under ultimate loads, actual volume of water
V_a	:	Premeasured quantity of water
V_b	:	Balance qantity of water left
Vc	:	Ultimate shear resistance of a concrete section
V_{co}	:	Ultimate shear resistance of a concrete section uncracked in flexure
V_p	:	Volume of sheathing sample used in water less study
V_{cr}	:	Ultimate shear resistance of a concrete section, cracked in flexure
V_{to}	:	Torsional shear stress a concrete section upto which no torsional reinforcement is required
V_t	:	Torsional shear stress at a section
V_{tu}	:	Ultimate torsional shear stress at a section
x	:	Distance in metre between points of operation of σ_{po} and $\sigma_{po}\ (x)$
X	:	Smaller dimension of links measured between centres of legs
y	:	Tensile fibre distance from the centroid of the concrete section
Y_{po}	:	$\dfrac{\text{Side of loaded area of end block}}{2}$
Yo	:	$\dfrac{\text{Side of end block}}{2}$
Y_1	:	Larger dimension of links measured between centres of legs
$\triangle$	:	Deviation of individual test strength from the average test strength of 'n' test strength results
μ	:	Coefficient of friction between cable and duct
θ	:	Cumulative angle in radian through which the tangent to the cable profile has turned between the points of operation of σ_{po} and $\sigma_{po}\ (x)$
σ	;	Standard deviation (population)
σ_{po}	:	Steel stress at the jacking end
$\sigma_{po}\ (x)$	:	Steel stress at a point, distant 'x' from the jacking end
ϕ	:	Internal nominal diameter of sheathing

1. INTRODUCTION

The object of issuing the Design Criteria for Prestressed Concrete Road Bridges by the Indian Roads Congress is to establish a common procedure for the design and construction of road bridges in India. This publication is meant to serve as a guide to both the design engineer and construction engineer but compliance with the provisions therein does not relieve them in any way of their responsibility for the stability and soundness of the structures designed and erected by them.

The design and construction of road bridges require an extensive and thorough knowledge of science and technique involved and should be entrusted only to specially qualified engineers with adequate experience of bridge engineering and capable of ensuring careful execution of work.

The requirement of revising the Design Criteria for Prestressed Concrete Road Bridges published in 1965 to cope with the technological developments which have taken place in this field of engineering has been felt for quite some time. A Subcommittee was accordingly constituted by the Bridges Committee in June, 1983. The Subcommittee consisting of the following personnel, finalised the revised Design Criteria for Prestressed Concrete Road Bridges at their meeting held from 22nd to 24th July, 1985 at New Delhi.

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This draft was considered by the Bridges Committee at their meeting held at New Delhi on the 20th August, 1985 and approved subject to some modifications, which was later approved by the Executive Committee on the 22nd August, 1985 at New Delhi and then by the Councill in their 114th meeting held on the 6th September, 1985 at Panaji, Goa, taking into consideration the views expressed by the members of the Council.

Wherever the provisions of these Criteria are different from the provisions of existing IRC:21-1972, IRC:22-1966, etc., the provisions of this Criteria shall be adopted. Wherever a reference to IS Code has been made, the latest revision shall be followed.

2. SCOPE

These Criteria cover the design aspects for prestressed concrete (post-tensioned) road bridges (determinate structures only). These are not applicable to the design of members which are subjected to direct compression like piers.

3. MATERIALS

3.1. Cement

Any of the following shall be used with prior approval of the competent authority :

(a) Ordinary Portland Cement conforming to IS:269

(b) Portland Slag Cement conforming to IS:455 but with not more than 50 per cent slag content

(c) Rapid Hardening Portland Cement conforming to IS:8041

(d) High Strength Portland Cement conforming to IS:8112

3.2. Aggregates

3.2.1. Coarse aggregate

3.2.1.1. Coarse aggregate shall consist of clean, hard, strong, dense non-porous and durable pieces of crushed stone, crushed gravel, natural gravel or a suitable combination thereof or other approved inert material. It shall not contain pieces of disintegrated stones, soft, flaky elongated particles, salt alkali, vegetable matter or other deleterious materials in such quantities as to reduce the strength or durability of the concrete, or to attack the embedded steel. It shall comply with IS:383.

The nominal maximum size of aggregates shall usually be restricted to 10 mm less than the minimum clear distance between individual cables or individual untensioned steel reinforcement or

10 mm less than the minimum cover to untensioned steel reinforcement, whichever is smaller. A nominal size of 20 mm coarse aggregates shall generally be considered satisfactory for prestressed concrete work.

3.2.2. **Fine aggregates :** Fine aggregates shall consist of hard, strong, durable, clean particles of natural sand, crushed stone or crushed gravel or suitable combination of natural sand and crushed stone or gravel. They shall not contain dust, lumps, soft or flaky materials, mica and other deleterious materials in such quantities as would reduce the strength or durability of concrete or attack the embedded steel. Fine aggregates shall conform to IS:383.

3.3. Water

Water used for mixing and curing shall be clean and free from injurious amounts of oils, acids, alkalis, salts, sugar, organic materials or other substances that may be deleterious to concrete or steel. Potable water is generally considered satisfactory for mixing concrete. As a guide the following concentrations represent the maximum permissible values :

(a) To neutralise 200 ml sample of water using phenolphthalein as an indicator, it should not require more than 2 ml of 0.1 normal NaOH.

(b) To neutralise 200 ml sample of water, using methylorange as an indicator, it should not require more than 10 ml of 0.1 normal HCl.

(c) Permissible limits for solids shall be as given below :

	Permissible limit (Max.)
Organic	200 mg/lit
Inorganic	3000 mg/lit
Sulphate (as SO_4)	500 mg/lit
Chlorides (Cl)	500 mg/lit
Suspended matter	2000 mg/lit

(d) The pH value, shall generally be not less than 6. Whenever necessary, tests shall be done as per IS: 3025. Mixing and curing with sea or creek water shall not be permitted.

3.4. Admixture may be provided in conformity with Clause 4.4. of IS : 1343.

3.5. Steel

3.5.1. The prestressing steel shall be any of the following :

(a) Plain hard drawn steel wire conformiig to IS:1785 (Part I) and IS:1785 (Part II)

(b) Cold drawn indented wire conforming to IS:6003
(c) High tensile steel bar conforming to IS: 2090 and
(d) Uncoated stress relieved strand conforming to IS: 6006

3.5.2. **Untensioned steel :** Reinforcement used as untensioned steel shall be any of the following :

(a) Mild steel and medium tensile steel bars conforming to IS:432 (Part I)
(b) Hot rolled deformed bars conforming to IS: 1139
(c) Cold twisted bars conforming to IS: 1786 and
(d) Hard drawn steel wire fabric conforming to IS: 1566

3.6. **Sheathing Strips**

3.6.1. Unless otherwise specified, the material shall be Cold Rolled Cold Annealed (CRCA) Mild Steel intended for mechanical treatment and surface refining but not for quench hardening or tempering.

3.6.2. The material shall normally be bright finished. However, in case of use in aggresive environment galvanised or lead coated mild steel strips shall be adopted.

3.6.3. The thickness of the strips shall be minimum of 0.24 mm ± 0.02 mm for internal diameter of sheathing ducts upto and including 51 mm and shall be 0.30 mm ± 0.02 for diameters beyond 51 mm and upto 91 mm.

3.6.4. The sheathing shall conform to the requirements as per tests specified in *Appendix 1* and a test certificate shall be furnished by the manufacturer.

4. CONCRETE

4.1. **Concrete Mix Design**

4.1.1. The concrete shall be designated by indicating the specified characteristic compressive strength of 150 mm cubes at 28 days expressed in MPa.

4.1.2. The characteristic compressive strength of concrete shall be defined as the strength of the concrete, below which not more than 5 per cent of the test results are expected to fall. For prestressed concrete work, the specified characteristic compressive strength shall not be less than 35 MPa i.e. grade M35 except for composite construction where concrete of grade M30 could be permitted for deck slab. The design shall be based on characteristic compressive strength of concrete.

4.1.3. The determination of the proportion of cement, aggregates and water to attain required strengths shall be made by designing the concrete mix. Such concrete shall be called "Design Mix Concrete". For prestressed concrete construction only "Design Mix Concrete" shall be used. The concrete mix shall be designed as per IS: 10262 (Recommended Guidelines for Concrete Mix Design) to have a target mean strength defined as $f_{ck} + 1.65\sigma$.

4.1.4. The concrete mix shall be designed for values of target mean strength not lower than those indicated in Table 1.

TABLE 1

Grade of concrete	Target mean strength MPa
M-35	47
M-40	52
M-45	58
M-50	63
M-55	69

4.1.5. The concrete mix shall also comply with the minimum cement content for different exposures as given in Table 2 and at the same time the quantity of cement in the concrete mix shall not be more than 540 kg/cum of concrete.

TABLE 2

Exposure		Minimum cement content (kg/cum)	Max. water cement ratio
Severe:	For example, exposed to saline atmosphere or freezing whilst wet or corrosive fumes	400	0.45
Other than severe:		360	0.50

Note : The minimum cement content is based on 20 mm nominal maximum size aggregates.

4.2. **Sampling and Testing for Strength**

4.2.1. For sampling and testing, statistical approach shall be followed. Samples from fresh concrete shall be taken as per

IS: 1199 and cubes shall be made cured and tested in accordance with IS: 516 subject to (a) and (b) below :-

(a) A random sampling procedure shall be adopted to ensure that each concrete batch shall have a reasonable chance of being tested. The sampling should, therefore, be spread over the entire period of concreting and cover all mixing units. The minimum frequency of sampling of concrete of each grade shall be one cube for every 2 m^3 of concrete for the first 300 m^3 of concrete or concrete in the first major span of bridge whichever is less, to be reduced to 1 cube for every 3 m^3 for subsequent work.

(b) Test cubes shall be made from each sample for testing at 28 days. Additional cubes may also be taken to determine the strength of concrete at any day other than 28 days to verify the strength at the time of transfer of prestress.

4.2.2. **Testing**

(a) Concrete of each grade shall be analysed separately to determine its standard deviation.

(b) Total number of test strength results required for calculation of standard deviation shall be preferably not less than 40. Attempts shall be made to obtain 40 test strength results as early as possible when a mix is used for the first time. When significant changes in the materials used, mix proportioning, equipment or technical control are made in the production of concrete, mix shall be redesigned. The standard deviation of concrete of a given grade shall be calculated from the individual test strength results using the following formula:

$$\text{Estimated Standard Deviation } \delta = \sqrt{\frac{\Sigma\Delta^2}{n-1}}$$

where Δ = deviation of individual test strength from the average test strength of 'n' test strength results

n = number of test strength results

4.3. **Acceptance Criteria**

4.3.1. The concrete shall be deemed to comply with the strength requirement if,

(a) every result has a strength not less than the design characteristic value, or

(b) the strength of one or more test results though less than the characteristic value is, in each case, not less than the greater of:

(1) The characteristic strength minus 1.35 times the standard deviation; and

(2) 0.80 times the characteristic strength;

and the average strength of all the samples is not less than the characteristic strength plus

$\left(1.65 - \frac{1.65}{\sqrt{n}}\right)$ times the standard deviation.

4.3.2. The concrete shall be deemed not to comply with the strength requirements if,

(a) The strength of any test result is less than the greater of:

(1) The characteristic strength minus 1.35 times the standard deviation; and

(2) 0.8 times the characteristic strength; or

(b) The average strength of all the test results is less than the characteristic strength plus $\left(1.65 - \frac{3}{\sqrt{n}}\right)$ times the standard deviation.

4.3.3. Concrete of each grade shall be assessed separately. Concrete of each identifiable unit of bridge or concrete cast per day shall be assessed for compliance.

4.3.4. Concrete which does not meet the strength requirements as specified in 4.3.1. but has a strength greater than that required by 4.3.2. may, at the discretion of the designer, be accepted as being structurally adequate without further testing.

If the concrete is deemed not to comply with 4.3.2., the structural adequacy of the parts affected shall be investigated and any consequential action, if needed, shall be taken.

5. LOADS AND FORCES

5.1. The loads and forces and load combinations as per IRC: 6-1966 and as applicable for the given structure shall be duly accounted for.

5.2. All critical loading stages shall be investigated. The stages stated below may normally be investigated:

(i) Stage prestressing;

(ii) Construction stages including temporary loading, transport, handling

and erection or any occasional loads that may occur during launching of girders, etc. including impact, if any;.

(iii) The design loads as per load combination of 5.1. above including the following discrete stages:

(a) Service Dead Load+Prestress with full losses.

(b) Service Dead Load+Live Load+Prestress with full losses.

(iv) Ultimate load, as per Clause 12.

6. STAGE PRESTRESSING

Stage prestressing is permissible. However, concrete shall have attained a strength of not less than 20 MPa before any prestressings is applied.

7. PERMISSIBLE STRESSES IN CONCRETE

7.1. Permissible Temporary Stresses in Concrete

7.1.1. These stresses are calculated after accounting for all losses except due to residual shrinkage and creep of concrete.

7.1.2. The compressive stress produced due to loading mentioned in clause 5.2. (ii) shall not exceed $0.5 f_{cj}$ which shall not be more than 20 MPa, where f_{cj} is the concrete strength at that time subject to a maximum value of f_{ck}.

7.1.3. At full transfer the cube strength of concrete shall not be less than $0.8\ f_{ck}$. Temporary compressive stress in the extreme fibre of concrete (including stage prestressing) shall not exceed $0.45\ f_{cj}$. subject to a maximum of 20 MPa.

7.1.4. The temporary tensile stresses in the extreme fibres of concrete shall not exceed 1/10th of the permissible temporary compressive stress in the concrete.

7.2. Permissible Stress in Concrete during Service

7.2.1. The compressive stress in concrete under service loads shall not exceed $0.33 f_{ck}$.

7.2.2. No tensile stress shall be permitted in the concrete during service.

7.2.3. If pre-cast segmental elements are joined by prestressing, the stresses in the extreme fibres of concrete during service shall always be compressive and the minimum compressive stress

in an extreme fibre shall not be less than five per cent of maximum permanent compressive stress that may be developed in the same section. This provision shall not, however, apply to cross prestressed deck slabs.

7.3. Permissible Bearing Stress Behind Anchorages

The maximum allowable stress, immediately behind the anchorages in adequately reinforced end blocks may be calculated by the equation :

$$f_b = 0.48 f_{cj} \sqrt{\frac{A_2}{A_1}} \text{ or } 0.8 f_{cj} \text{ whichever is smaller}$$

where

f_b= the permissible compressive contact stress in concrete including any prevailing stress as in the case of intermediate anchorages

A_1 = the bearing area of the anchorage converted in shape to a square of equivalent area

A_2 = the maximum area of the square that can be contained within the member without overlapping the corresponding area of adjacent anchorages, and concentric with the bearing area A_1.

Notes : (i) The above value of bearing stress is permissible only if there is a projection of concrete of at least 50 mm or $b_{1/4}$, whichever is more all round the anchorage, where b_1 is as shown in Fig. 1.

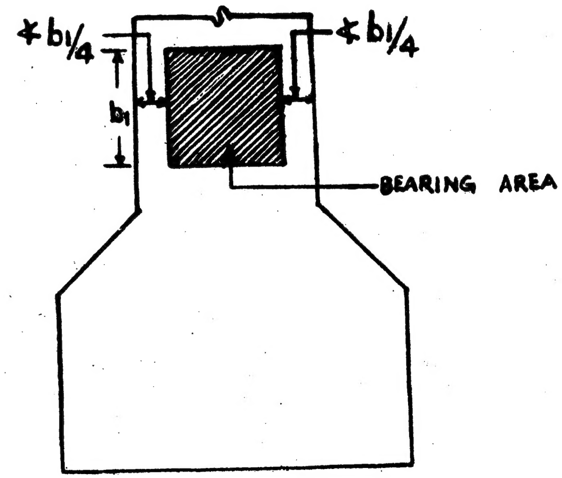


Fig. 1

(ii) The value of f_b, so calculated, may be increased suitably, if adequate hoop reinforcement is provided at the anchorage.

(iii) When anchorages are embedded in concrete, the bearing stress shall be investigated after accounting for the surface friction between the anchorage and the concrete.

(iv) The pressure operating on the anchorage shall be taken before allowing for losses due to creep and shrinkage of concrete, but after allowing for losses due to elastic shortening, relaxation of steel and seating of anchorage.

8. PERMISSIBLE STRESSES IN PRESTRESSING STEEL

8.1. Permissible Stress

The maximum temporary stress in the prestressing steel at any section after allowing for losses due to slip of anchorages and elastic shortening of the member should not exceed 70 per cent of the minimum ultimate tensile strength as specified in the relevant Indian Standard Code.

8.2. Overstressing

Overstressing of prestressing steel to compensate for slipping of anchorages or to achieve the calculated extensions may be permitted subject to the jacking force not exceeding 80 per cent of the minimum ultimate tensile strength or 95 per cent of the proof stress (0.2 per cent) of the prestressing steel whichever is less.

9. SECTION PROPERTIES

9.1. For members consisting of precast as well as cast-in-situ units, due consideration shall be given to the different moduli of elasticity of concrete in the precast and in-situ portions.

9.2. Openings in Concrete Section

For the purpose of determining the flexural stresses both prior to and after grouting of the cables or tendons, the properties of the section such as area, position of centroid and moment of inertia may be based upon the full section of the concrete without deducting for the area of longitudinal openings left in the concrete for prestressing tendons, cable ducts or sheaths. No allowance for the transformed area of the prestressing tendons shall, however, be made.

Deduction shall be made for the holes of transverse prestressing tendons at sections where they occur, for determining the stresses before grouting of these holes.

9.3. Minimum Dimensions

9.3.1. 'T' Beams

9.3.1.1. The thickness of the web shall not be less than 150 mm + diameter of duct hole. Where cables cross within the web, suitable increase in the thickness over the above value shall be made.

9.3.1.2. The effective width of the flange of a 'T' beam shall conform to clause 305.12.2. of IRC : 21.

9.3.1.3. The minmum thickness of the deck slab within the webs shall be 150 mm for normal conditions of exposure. Under aggressive conditions of exposure the minimum thickness shall be 180 mm. However, beyond the outer webs, the thickness may be gradually reduced to have a minimum thickness of 150 mm at the tip of the cantilever.

9.3.2. Box Girders

9.3.2.1. The thickness of the web shall not be less than d/36 + twice the clear cover to the reinforcement + diameter of the duct hole where 'd' is the overall depth of the beam measured from the top of the deck slab to the bottom of the beam or 150 mm + diameter of the duct holes, which ever is greater. Where ever cables cross within the web, suitable increase in the thickness over the above value shall be made.

9.3.2.2. The thickness of the bottom flange of box girder shall be not less than 1/30th of the clear web spacing at the junction with bottom flange or 150 mm which ever is more. For top flange minimum thickness shall be as per clause 9.3.1.3.

9.3.2.3. For top and bottom flange having prestresssing cables, the thickness of such flange shall not be less than 150 mm plus diameter of duct hole.

9.3.2.4. In box girders, effective and adequate bond and shear resistance shall be provided at the junction of the web and the slabs. The slabs may be considered as an integral part of the girder and the entire width may be assumed to be effective in compression.

For very short spans or where web spacing is excessive or where overhangs are excessive, analytical investigation shall be made to determine the effective flange width.

9.3.2.5. Haunches of minimum size of 300 mm (horizontal) and 150 mm (vertical) shall be provided at the four extreme inner corners of the box section. For all other corners fillets of suitable size may be provided.

9.4. Diaphragms/Cross Girders

Diaphragms shall be provided depending upon design requirements. The thickness of diaphragms shall not be less than the minimum web thickness.

10. MODULI OF ELASTICITY

10.1. Modulus of Elasticity of Steel (E_s)

10.1.1. For the purpose of design the following nominal values of modulus of elasticity can be assumed except where the manufacturers certified values or test results are available:

TABLE 3

Type of steel	Modulus of elasticity MPa
Plain hard-drawn wires (conforming to IS:1785 and IS: 6003)	2.1×10^5
High tensile steel bars rolled or heat treated (conforming to IS: 2090)	2.0×10^5
Strands (conforming to IS:6006)	1.95×10^5

10.1.2. Representative values of modulus of elasticity as supplied by the manufacturers or as per test results based on one test of 3 samples for every lot of 10 tonnes or part thereof shall be used for verification of the elongation calculations.

10.2. Modulus of Elasticity of Concrete (E_c)

Unless otherwise determined by tests, the modulus of elasticity, E_c of concrete shall be assumed to have a value

$E_c = 5700 \sqrt{f_{ck}}$ MPa

The value of the modulus of elasticity E_{cj} of the concrete at j days may be taken to be.

$E_{cj} = 5700\sqrt{f_{cj}}$

11. LOSSES IN PRESTRESS

Decrease in prestress in steel due to elastic shortening, creep

and shrinkage of concrete, relaxation of steel, friction and seating of anchorages shall be calculated on the following basis:

11.1. Elastic Shortening

The loss due to elastic shortening of concrete shall be computed based on the sequence of tensioning. However, for design purposes, the resultant loss of prestress in tendons tensioned one by one may be calculated on the basis of half the product of modular ratio and the stress in concrete adjacent to the tendons averaged along the length. Alternatively the loss of prestress may be computed exactly based on sequence of stressing.

11.2. Creep of Concrete

The strain due to creep of concrete shall be taken as specified in Table 4.

TABLE 4

Maturity of concrete at the time of stressing as a percentage of f_{ck}	Creep strain per 10 MPa
40	9.4×10^{-4}
50	8.3×10^{-4}
60	7.2×10^{-4}
70	6.1×10^{-4}
75	5.6×10^{-4}
80	5.1×10^{-4}
90	4.4×10^{-4}
100	4.0×10^{-4}
110	3.6×10^{-4}

Notes :

(i) The creep strain during any interval may be taken as the strain due to a sustained stress equal to the arithmetic mean of the initial and the final stress occurring during that interval.

(ii) The stress for the calculation of the loss due to creep shall be taken as the stress in concrete at the centroid of the prestressing steel. Variation in stress, if any, along the centroid of the prestressing steel, may be accounted for.

(iii) Values of creep strain for intermediate figures for the maturity of concrete at the time of stressing may be interpolated taking a linear variation between the values given above.

(iv) The above values are for Ordinary Portland Cement.

11.3. Shrinkage of Concrete

The loss in prestress in steel, due to shrinkage of concrete shall be estimated from the values of strain due to residual shrinkage given in Table 5.

TABLE 5

Age of concrete at the time of stressing, in days	Strain due to residual shrinkage
3	4.3×10^{-4}
7	3.5×10^{-4}
10	3.0×10^{-4}
14	2.5×10^{-4}
21	2.0×10^{-4}
28	1.9×10^{-4}
90	1.5×10^{-4}

Notes: (i) Values for intermediate figures for any age of concrete may be interpolated taking a linear variation between the values given.

(ii) The above are for Ordinary Portland Cement.

11.4. Relaxation of Steel

When certified values are not available, the relaxation losses may be assumed as given in Table 6.

TABLE 6

Initial stress	Relaxation loss MPa
0 5 fp	0
0.6 fp	35
0.7 fp	70
0.8 fp	90

Notes : (i) For intermediate values linear interpolation may be done

(ii) fp = U. T. S.of steel

11.5. Losses Due to Seating of Anchorages

Depending upon the type of post tensioning, losses in prestress occur due to slip of wires, draw-in of male cones, strains in anchorage, the value of which shall be as per tests or manufacturer's recommendations and duly accounted, for considering reverse friction near the anchorage ends. For this purpose the values of co-efficient of friction and wobble co-efficient shall be taken same as those stipulated for positive friction.

11.6. Friction Losses

Steel stress in prestressing tendons $\sigma_{po\,(x)}$ at any distance x from the jacking end can be calculated from the formula

$$\sigma_{po} = \sigma_{po\,(x)}\, e^{(\mu\theta + kx)}$$

where

σ_{po}	=	the steel stress at the jacking end
e	=	the base of Naperian Logarithms
μ	=	the co-efficient of friction
θ	=	the cumulative angle in radians through which the tangent to the cable profile has turned between the points of operation of σ_{po} and $\sigma_{po}\,(x)$.
$\sigma_{po\,(x)}$	=	the steel strees at a point, distant 'x' from the jacking end
k	=	the wobble co-efficient per metre length of steel
x	=	the distance between points of operation of σ_{po} and $\sigma_{po(x)}$ in metres.

The values of μ and k given in Table 7 may be adopted for calculating the friction losses.

TABLE 7

Type of high tensile steel	Type of duct or sheath	Values recommended to be used in design	
		k per metre	μ
Wire cables	Bright metal	0.0091	0.25
	Galvanized	0.0046	0.2
	Lead coated	0.0046	0.18
	Unlined duct in concrete	0.0046	0.45
Uncoated stress relieved strands	Bright metal	0.0046	0.25
	Galvanized	0.0030	0.20
	Lead coated	0.0030	0.18
	Unlined duct in concrete	0.0046	0.50

Notes: (i) Values to be used in design may be altered to the values observed, on satisfactory evidence in support of such values.

(ii) For multi-layer wire cables with spacer plates providing lateral separation, the value of μ may be adopted on the basis of actual test results.

(iii) When the direction of friction is reversed, the index of 'e' in the above formula shall be negative.

(iv) The above formula is of general application and can be used for estimation of friction between any two points along the tendon distant 'x' from each other.

The values of μ and k used in design shall be indicated on the drawings for guidance in selection of the material and the methods that will produce results approaching the assumed values.

12. ULTIMATE STRENGTH

A prestressed concrete structure and its constituent members shall be checked for failure conditions at an ultimate load of $1.25\,G + 2\,SG + 2.5\,Q$ under normal condition and $1.5\,G + 2\,SG + 2.5\,Q$ under severe exposure conditions where G, SG and Q denote permanent load, superimposed dead load and live load including impact respectively. The superimposed dead load shall include dead load of precast footpath, hand rails, wearing course, utility services kerbs etc. For sections, where the dead load causes effects opposite to those of live load, the sections shall also be checked for adequacy for a load of $G + SG + 2.5\,Q$.

13. CALCULATION OF ULTIMATE STRENGTH

Under ultimate load conditions, the failure may either occur by yielding of the steel (under-reinforced section) or by direct crushing of the concrete (over-reinforced section). The section should be so proportioned that M_{ult} obtained by yielding of steel is less than that obtained by crushing of concrete. Ultimate moment of resistance of sections, under these two alternative conditions of failure shall be calculated by the following formulae :

(i) Failure by yield of steel (under-reinforced section)

$$M_{ult} = 0.9\, d_b\, A_s\, f_p$$

where

A_s = the area of high tensile steel

f_p = the ultimate tensile strength for steel without definite yield point or yield stress or stress at 4 per cent elongation whichever is higher for steel with a definite yield point.

d_b = the depth of the beam from the maximum compression edge to the centre of gravity of the steel tendons.

Non-prestressed reinforcement may be considered as contributing to the available tension for calculation of the ultimate moment of resistance in an amount equal to its area times its yield stress, provided such reinforcement is welded or has sufficient bond under conditions of ultimate load.

(ii) Failure by crushing of concrete

$$M_{ult} = 0.176\, b d_b^2 f_{ck}, \text{ for a rectangular section.}$$

$$M_{ult} = 0.176\, b d_b^2 f_{ck} + \frac{2}{3}\, 0.8\, (B_f - b)\left(d_b - \frac{t}{2}\right) \times t f_{ck}$$

for a Tee beam.

where

b = the width of rectangular section or web of a Tee beam

B_f = the width of flange of Tee beam.

t = the thickness of flange of a Tee beam.

14. SHEAR AND TORSION

14.1. Shear

14.1.1. The calculations for shear are only required for the Ultimate Load.

At any section the ultimate shear resistance of the concrete alone, V_c shall be considered for the section both uncracked (see 14.1.2.) and cracked (see 14.1.3.) in flexure, and the lesser value taken and, if necessary, shear reinforcement (see 14.1.4.) provided.

For a cracked section, the conditions of maximum shear with co-existent bending moment and maximum bending moment with co-existent shear shall both be considered.

The effect of the vertical component of the bottom flange force in members of variable depth may also be considered where applicable. While calculating this component the design moment to be considered shall be concomitant with the design shear force being considered.

14.1.2. **Sections uncracked in flexure :** The ultimate shear resistance of a section uncracked in flexure, V_{co}, corresponds to the occurrence of a maximum principal tensile stress, at the centroidal axis of the section, of $f_t = 0.24\sqrt{f_{ck}}$.

In the calculation of V_{co}, the value of prestress at the centroidal axis has been taken as 0.8 f_{cp}. The value of V_{co} is given by :

$$V_{co} = 0.67\ bd\ \sqrt{f_t^2 + 0.8 f_{cp} f_t}$$

where

*b = Width in the case of rectangular member and width of the rib in the case of T, I and L beams

d = overall depth of the member

f_t = maximum principal tensile stress given by $0.24\sqrt{f_{ck}}$

f_{cp}= compressive stress at centroidal axis due to prestress taken as positive.

*Where the position of a duct coincides with the position of maximum principal tensile stress, e.g., at or near the junction of flange and web near a support, the value of b should be reduced by the full diameter of the duct if ungrouted and by two-thirds of the diameter if grouted.

In flanged members where the centroidal axis occurs in the flange, the principal tensile stress should be limited to $0.24\sqrt{f_{ck}}$ at the intersection of the flange and web; in this calculation, 0.8 of the stress due to prestress at this intersection may be used in calculating V_{co}

For a section uncracked in flexure and with inclined tendons or vertical prestress, the component of prestressing force normal to the longitudinal axis of the member may be added to V_{co}.

14.1.3. **Sections cracked in flexure :** The ultimate shear resistance of a section cracked in flexure, V_{cr} may be calculated using the equation given below :

$$V_{cr} = 0.037\ bd_b\ \sqrt{f_{ck}} + \frac{M_t}{M} V$$

where

d_b = is the distance from the extreme compression fibre to the centroid of the tendons at the section considered;

M_t is the cracking moment at the section considered. $M_t = (0.37\sqrt{f_{ck}} + 0.8 f_{pt}) I/y$ in which f_{pt} is the stress due to prestress only at the tensile fibre distance y from the centroid of the concrete section which has a second moment of area I;

V and M are the shear force and corresponding bending moment at the section considered due to ultimate loads;

V_{cr} should be taken as not less than $0.1\ bd\sqrt{f_{ck}}$. The value of V_{cr} calculated at a particular section may be assumed to be constant for a distance equal to $d_b/2$, measured in the direction of increasing moment from that particular section.

For a section cracked in flexure and with inclined tendons, the component of prestressing forces normal to the longitudinal axis of the member should be ignored.

14.1.4. **Shear reinforcement:** When V, the shear force due to the ultimate load is less than $V_c/2$ then no shear reinforcement need be provided. A minimum shear reinforcement shall be provided when V is greater than $V_c/2$ in the form of links such that

$$\frac{A_{sv}}{S_v} \times \frac{0.87 f_{yv}}{b} = 0.4 \text{ MPa} \qquad \dots (1)$$

When the shear force V, due to the ultimate loads exceeds V_c, the shear reinforcement provided shall be such that

$$\frac{A_{sv}}{S_v} = \frac{V - V_c}{0.87 f_{yv} d_t} \qquad \dots (2)$$

Where V_c is the shear force that can be carried by the concrete

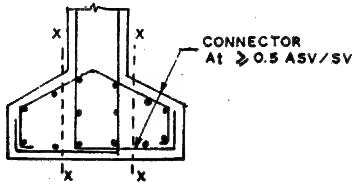
f_{yv} is the yield strength of the reinforcement or 0.2 per cent proof stress which should be taken as not greater than 415 MPa

A_{sv} is the cross-sectional area of the two legs of a link

S_v is the link spacing along the length of member

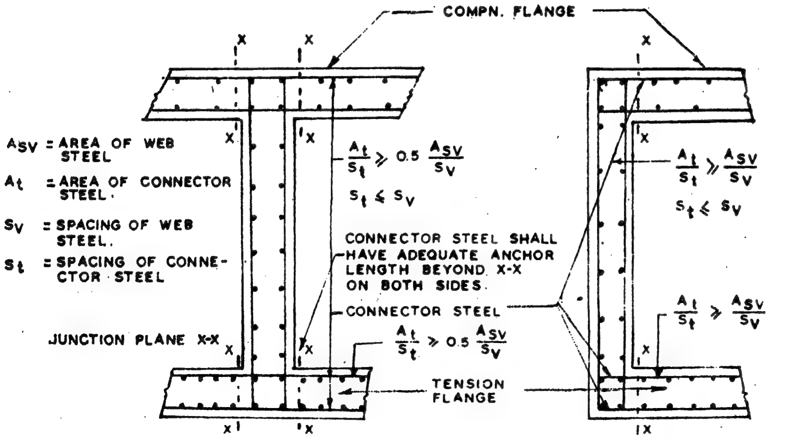
d_t is the depth from the extreme compression fibre either to the longitudinal bars or to the centroid of the tendons whichever is greater.

In beams, at both corners in the tensile zone, a link shall pass round a longitudinal bar, a tendon or a group of tendons having a diameter not less than the link diameter. A link shall extend as close to the tension and compression faces as possible, with due regard to cover. The links provided at a cross section shall between them enclose all the tendons and additional reinforcement provided at the cross section and shall be adequately anchored, Fig.2 (i-iii). In no case shear reinforcement provided shall be less than that required as per equation (1) above when V is greater than $V_c/2$.



(i) Connector details tension flange (bulb) of T-beam

N.B. Connector steel shall have adequate anchorage length beyond 'XX' on both sides.



(ii) Connector details at junction of intermediate web and flanges of box girder

(iii) Connector details at junction of end webs and flanges of box girder

Fig. 2

14.1.5. **Maximum shear force:** In no circumstances shall the shear force 'V', due to ultimate loads, exceed the appropriate value given in Table 8 multiplied by bd_b, where 'b' is as defined in sub-clause 14.1.2, less either the diameter of the duct for ungrouted or two-thirds the diameter of the duct for grouted ducts and 'd_b' is the distance from the compression face to the centroid of the actual steel area in tensile zone.

The shear force V should include an allowance for prestressing only for sections uncracked in flexure (see 14.1.2).

TABLE 8. MAXIMUM SHEAR STRESS

	Concrete Grade				
	30	35	40	50	55 and over
	MPa	MPa	MPa	MPa	MPa
Maximum Shear Stress	4.1	4.4	4.7	5.3	5.5

Note : For intermediate values linear interpolation may be done.

14.2. Torsional Resistance of Beams

14.2.1. **General :** Torsion does not usually decide the dimensions of members, therefore, torsional design should be carried out as a check after the flexural design. In general, where the torsional resistance or stiffness of members has not been taken into account in the analysis of the structure no specific calculations for torsion will be necessary, adequate control of any torsional cracking being provided by the required nominal shear reinforcement. Therefore, provisions made in the clause are to be followed when the effect of torsion is appreciable. Alternative methods of designing members subjected to combined bending, shear and torsion could also be used provided the rationality of the method adopted is justified.

14.2.2. **Stresses and reinforcement :** Calculations for torsion are required only for ultimate loads and the torsional shear stresses should be calculated assuming a plastic stress distribution. Where the torsional shear stress Vt, exceeds the value V_{to} from Table 9, reinforcement shall be provided. In no case, shall the sum of shear stresses resulting from shear force and torsion $(V + Vt)$ exceed the value of V_{tu} from Table 9 nor in the case of

small sections ($Y_1 < 550$ mm) should the torsional shear stress, V_t, exceed $V_{tu} \times Y_1/550$ where Y_1 is the larger dimension of a link.

Torsion reinforcement shall consist of rectangular effectively closed links together with longitudinal reinforcement. This reinforcement is addition to that required for shear or bending.

TABLE 9. ULTIMATE TORSION SHEAR STRESS

	Concrete Grade		
	30	35	40 or above
	MPa	MPa	MPa
V_{tc}	0.37	0.40	0.42
V_{tu}	4.10	4.45	4.75

14.2.3. **Computation of torsional stresses for various cross sections**

14.2.3.1. **Rectangular section**

$$V_t = \frac{2T}{\left(h^2_{min}\right)\left(h_{max} - h_{min}/3\right)}$$

where

T is the torsional moment due to ultimate loads

h_{min} is the smaller dimension of the section

h_{max} is the larger dimension of the section

Torsional reinforcement should be provided such that

$$\frac{A_{sv}}{S_v} \geqslant \frac{T}{0.8\, X_1\, Y_1\, (0.87 f_{yv})}$$

$$A_{SL} \geqslant \frac{A_{sv}}{S_v}(X_1 + Y_1)\left(\frac{f_{yv}}{f_{yL}}\right)$$

where

A_{sv} is the total area of legs of closed links at a section

A_{SL} is the area of longitudinal reinforcement

f_{yL} is the yield strength of longitudinal reinforcement which should not be taken greater than 415 MPa.

f_{yv} is the yield strength of links

S_v is the spacing of the links

X_1 is the smaller dimension of the link measured between centres of legs

Y_1 is the larger dimension of the link measured between centres of legs.

To prevent a detailing failure the closed links shall be detailed to have minimum cover and a pitch less than the smallest of $(X_1 \times Y_1)/4$, 16 times longitudinal corner bar diameters and 200 mm. The longitudinal reinforcement shall be positioned uniformly such that there is a bar at each corner of the links. The diameter of the corner bars shall be not less than the diameter of the links.

14.2.3.2. **T.L. and I sections** : Such sections shall be divided into component rectangles for purpose of torsional design. This shall be done in such a way as to maximize the function $\Sigma(h_{max} \times h^3_{min})$ where h_{max} and h_{min} are the larger and smaller dimensions of each component rectangle. Each rectangle shall then be considered subject to a torque :

$$\frac{T\,(h_{max} \times h^3_{min})}{\Sigma\,(h_{max} \times h^3_{min})}$$

Reinforcement shall be so detailed as to tie the individual rectangles together. Where the torsional shear stress in a rectangle is less than Vtc no torsional reinforcement need to be provided in that rectangle.

14.2.3.3. **Box section**

$$V_t = T/2h_{wo}\, A_o$$

where

h_{wo} is the wall thickness of members where the stress is determined;

A_o is the area enclosed by the centre line of members forming the box.

Torsional reinforcement is to be provided such that

$$\frac{A_{sv}}{S_v} \geqslant \frac{T}{A_o\,(0.87 f_{yv})}$$

$$A_{SL} \geqslant \frac{A_{sv}}{S_v}\left(\frac{\text{Perimeter of } A_o}{2}\right) \times \frac{f_{yv}}{f_{yl}}$$

The detailing requirements of 14.2.3.1. should still be observed. In detailing the longitudinal reinforcement to cater for torsional stresses account may be taken of those areas of the cross section subjected to simultaneous flexural compressive stresses and a lesser amount of reinforcement in the compressive zone may be taken as:

$$\text{Reduction in steel area} = f_{cav} \times \frac{\text{(Area of section subjected to flexural compression)}}{0.87\, f_{yc}}$$

Where f_{cav} is the average compressive stress in the flexural compressive zone and f_{yc} is the yield stress of longitudinal steel in compression.

15. MINIMUM REINFORCEMENT

15.1. General

The quantity of untensioned steel required for design or constructional purposes shall not be less than the minimum stipulated in clauses 15.2. to 15.4. Various types of minimum steel requirements need not be added together. The bars in such reinforcement shall, however, not be placed more than 200 mm apart, the diameter of mild steel and high strength deformed bars should not be less than 10 mm and 8 mm respectively.

15.2. In the vertical direction, a minimum reinforcement shall be provided in the bulb/web of the beams/rib of box girders, such reinforcement being not less than 0.3 per cent of the cross sectional area of the bulb/web in plan for mild steel and 0.18 per cent for HYSD bars respectively. Such reinforcement shall be as far as possible uniformly spaced along the length of the web. In the bulb portion, the cross sectional area of bulb in plan shall be taken.

In all the corners of the section, these reinforcements should pass round a longitudinal bar having a diameter not lesss than that of the vertical bar or round a group of tendons. For tee-beams, the arrangement in the bulb portion shall be as shown in Fig. 2.

15.3. Longitudinal reinforcements provided shall not be less than 0.25 per cent and 0.15 per cent of the gross cross sectional area of the section for mild steel and HYSD bars respectively, where the specified grade of concrete is less than M 45. In case the grade of concrete exceeds M45, the provision shall be increased

to 0.3 per cent and 0.18 per cent respectively. Such reinforcement shall as far as possible be evenly spaced on the periphery. Non-prestressed high tensile reinforcement can also be reckoned for the purpose of fulfilling the requirement of this clause.

15.4. For solid slabs and top and bottom slabs of box girders, the top and underside of the slabs shall be provided with reinforcement consisting of a grid formed by layers of bars. The minimum steel provided shall be as follows:

(i) For solid slabs and top slab of box girders: 0.3 per cent and 0.18 per cent of the gross cross sectional area of the slab for MS and HYSD bars respectively, which shall be equally distributed at top and bottom.

(ii) For soffit slab of box girders:
The longitudinal steel shall be at least 0.18 per cent and 0.3 per cent of sectional area for HYSD and MS bars respectively. The minimum transverse reinforcement shall be 0.3 per cent and 0.5 per cent of the sectional area for HYSD and MS bars respectively. The minimum reinforcement shall be equally distributed at top and bottom.

15.5. For cantilever slab minimum reinforcement of 4 nos. of 16 mm dia HYSD bars or 6 nos. of 16 mm dia MS bars should be provided with minimum spacing at the tip divided equally between the top and bottom surface parallel to support.

N.B. Notwithstanding the nomenclature "untensioned steel", this provision of reinforcement may be utilised for withstanding all action affects, if necessary.

16. COVER AND SPACING OF PRESTRESSING STEEL

16.1. The nominal clear cover measured from outside of sheathing shall be as follows:

16.1.1. For moderate conditions the clear cover shall be 50 mm.

16.1.2. For severe exposure conditions such as members located in coastal areas, members exposed to salt water, sea spray, chemical vapour, contact with earth, etc. the clear cover shall be 60 mm.

16.1.3. Clear cover shall be provided to untensioned reinforcement, including links and stirrups when using the indicated grade of concrete under particular condition of exposure as given in Table 10.

TABLE 10

Concrete Mix	Less than M 40	M 40 and above
Conditions of exposure	Nominal cover in mm	
Moderate	30	25
Severe	40	30

Notes (i) At each end of a reinforcing bar nominal cover shall not be less than twice the diameter of such bar.

(ii) For portions of structures in contact with water, where the velocity and bed material are likely to cause erosion of concrete the condition of exposure shall be assumed to be severe.

16.2. A minimum clear distance of 50 mm or 10 mm in excess of the largest size of aggregate used or diameter of the duct, whichever is greater, shall be maintained between individual cables when grouping of cables is not involved.

16.3. **Grouped Cables**

16.3.1. Grouping of cables shall be avoided to the extent possible. If unavoidable, only vertical grouping of cables, upto 2 cables may be permitted as shown in Fig. 3. The minimum clear spacing between groups shall be diameter of the duct or 50 mm or 10 mm in excess of the largest size of the aggregate, whichever is greater.

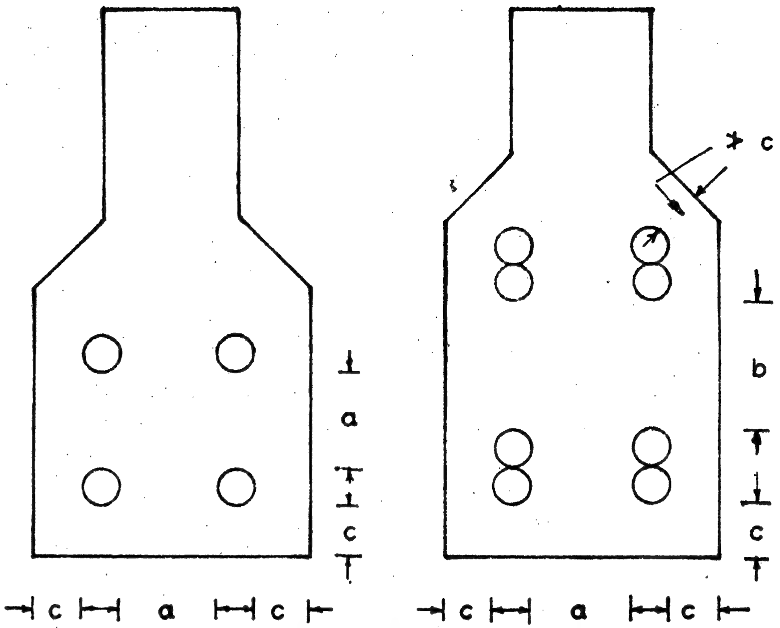
Note : In case of aggressive environment, grouping of cables should be altogether avoided. This may be achieved by the use of high capacity strands.

16.3.2. Individual cables or ducts of grouped cables shall be deflected or draped in the end portions of members. The clear spacing between cables or ducts in the end one metre of the members as specified in Clause 16.2. shall be maintained.

16.4. The placement of cables or ducts and the order of stressing and grouting shall be so arranged that the prestressing steel, when tensioned and grouted, does not adversely affect the adjoining ducts.

17. END BLOCKS

17.1. End block shall be designed to distribute the concentrated prestressing force at the anchorage. It shall have sufficient area to accommodate anchorages at the jacking end and shall preferably be as wide as the narrowest flange of the beam. Length of end block in no case shall be less than 600 mm nor less than its width. The portion housing the anchorages shall as far as possible be precast.



$a, b \not< 50$ mm or 10 mm in excess of the largest size of aggregate used or diameter of duct whichever is greater
$c \not< 50$ mm, the value of c to be increased by 10 mm for severe exposure conditions

Fig. 3

17.2. The bursting forces in the end blocks, should be assessed on the basis of the ultimate tensile strength. The bursting tensile force, F_{bst}, existing in an individual square end block loaded by a symmetrically placed square anchorage or bearing plate, may be derived from Table 11.

where :

$2\,Y_o$ is the side of end block

$2\,Y_{po}$ is the side of loaded area

P_k is the load in the tendon assessed as above

F_{bst} is the bursting tensile force.

TABLE 11. DESIGN BURSTING TENSILE FORCES IN END BLOCKS

Y_{po}/Y_o	0.3	0.4	0.5	0.6	0.7
F_{bst}/P_k	0.23	0.20	0.17	0.14	0.11

Notes : (i) For intermediate values linear interpolation may be made.

(ii) The values in the table above generally hold good for internal anchorages. For external anchorages the design force may be increased by 10 per cent.

This force, F_{bst}, will be distributed in a region extending from 0.2 Y_o to 2 Y_o from the loaded face of the end block as shown in Fig 4.

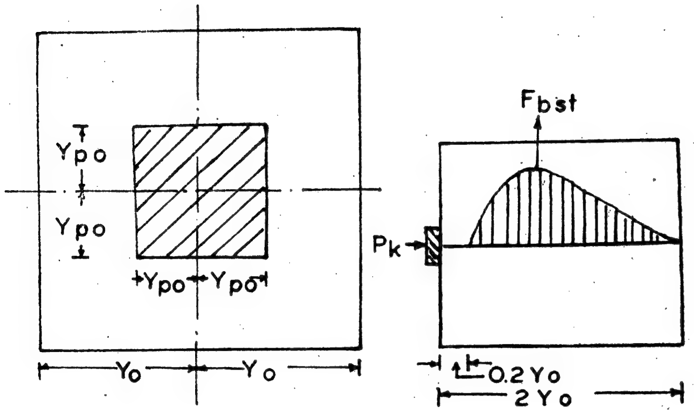


Fig. 4

Reinforcement provided in this region to sustain the bursting tensile force may be calculated based on a tensile strength of 0.87 f_p except that the stress should be limited to a value corresponding to a strain of 0.001 when the concrete cover to the reinforcement is less than 50 mm.

In the rectangular end blocks, the bursting tensile forces in the two principal directions should be assessed on the similar basis as in Table 11.

When circular anchorages or bearing plates are used, the side of the equivalent square area should be derived.

Where groups of anchorages or bearing plates occur, the end block should be divided into a series of symmetrically loaded prisms and each prism treated in the above manner. In detailing the reinforcement for the end block as a whole, it is necessary to ensure that the groups of anchorages are appropriately tied together. Special attention should be paid to end blocks having a cross section different in shape from that of the general cross section of the beam and reference should be made to specialist

literature. Compliance with the above requirements will generally ensure that bursting tensile forces along the loaded axis are provided for. In case where large concentrated tendon forces are involved alternative methods of design based on specialist literature and manufacturer's data may be more appropriate.

17.3. Consideration should also be given to the spalling tensile stresses that occur in end blocks. Where the anchorge or bearing plates are highly eccentric, these stresses reach a maximum at the loaded face. The end face of anchorage zone shall be continuously reinforced to prevent edge spalling. Reinforcement shall be placed as close to the end face as possible.

18. THICKENING OF WEBS OF GIRDERS

The thickening of webs of girders towards the end blocks shall be achieved gradually with a splay in plan of not more than 1 in 4. Suitable thickening for isolated anchorages away from the end blocks shall be made whenever necessary to reduce stress concentration.

19. INTERMEDIATE ANCHORAGES

The number of anchorages of cables/tendons which are stressed from the deck slab commonly known as intermediate anchorages shall be kept to the minimum possible with a view to prevent the possibility of ingress of water inside the cable through such intermediate anchorages during construction (before grouting). Suitable protective measures shall be adopted, such as providing sheathing around the cables/tendons projected beyond the anchorages and sealing the junction, filling the niches temporarily by lean concrete, threading the cable just before stressing, etc. Where intermediate anchorages are embedded in concrete, adequate reinforcement shall be provided to cater to local splitting forces and adverse stress conditions, that may develop.

20. SPLAY OF CABLES IN PLAN AND MINIMUM RADIUS OF CABLES IN ELEVATION

The splay of cables in plan, for bringing them from their position in the bottom flange at mid-span into the web towards the supports shall not be more than 1 in 6. The points of splay shall be suitably staggered on both sides of the longitudinal centre line of the web of the girder. In general, the radius of cables in elevation shall not be less than 1.5 m+700 d_s where d_s is the diameter of the wire or strand in mm as the case may be.

21. SLENDER BEAMS

The handling and erection of slender beams need special care. It is normally safe to erect such beams without reducing the permissible stress when the ratio of span to the width of top flange does not exceed 60 and further when the width of top flange is not less than one-fourth the depth of the beam. When the above conditions are not satisfied, adequate temporary restraints shall be provided and the permissible stresses reduced suitably.

22. EMERGENCY CABLES/STRANDS

Besides the design requirements, additional cables/strands shall be symmetrically placed in the structure so as to be capable of generating prestressing force of about 4 per cent of the total design prestressing force in the structure. Only those cables which are required to make up the deficiency shall be stressed and the remainder pulled out and the duct hole shall be grouted.

23. GROUTING OF CABLES

A recommended practice for grouting of cable is given at Appendix 2.

Appendix 1

TESTS ON SHEATHING DUCTS

1. All tests specified below shall be carried out on the same sample in the order given below.
2. At least 3 samples for one lot of supply (not exceeding 7000 metre length) shall be tested.
3. The tests are applicable for sheathing transported to site in straight lengths where the prestressing cable is threaded inside the sheathing prior to concreting. These tests are not applicable for sheathing and cable coiled and transported to site as an assembled unit, nor for sheathing ducts placed in position without threading of prestressing cable prior to concreting.

4. WORKABILITY TEST

A test sample 1100 mm long is soldered to a fixed base plate with a soft solder (Fig. 5). The sample is then bent to a radius of 1800 mm alternately on either side to complete 3 cycles.

Thereafter, the sealing joints will be visually inspected to verify that no failure or opening has taken place.

5. TRANSVERSE LOAD RATING TEST

The test ensures that stiffness of the sheathing is sufficient to prevent permanent distortion during site handling.

The sample is placed on a horizontal suppport 500 mm long so that the sample is supported at all points of outward corrugations.

A load as specifed in Table 12 is applied gradually at the centre of the supported portion through a circular contact surface of 12 mm ϕ. Couplers shall be placed so that the load is applied approximately at the centre of two corrugations, Fig. 6. The load as specified below is applied in increments.

TABLE 12

Dia :	25 mm to 35 mm	35 mm to 45 mm	45 mm to 55 mm	55 mm to 65 mm	65 mm to 75 mm	75 mm to 85 mm	85 mm to 90 mm
Load :	250 N	400 N	500 N	600 N	700 N	800 N	1000 N

The sample is considered acceptable if the permanent deformation is less than 5 per cent.

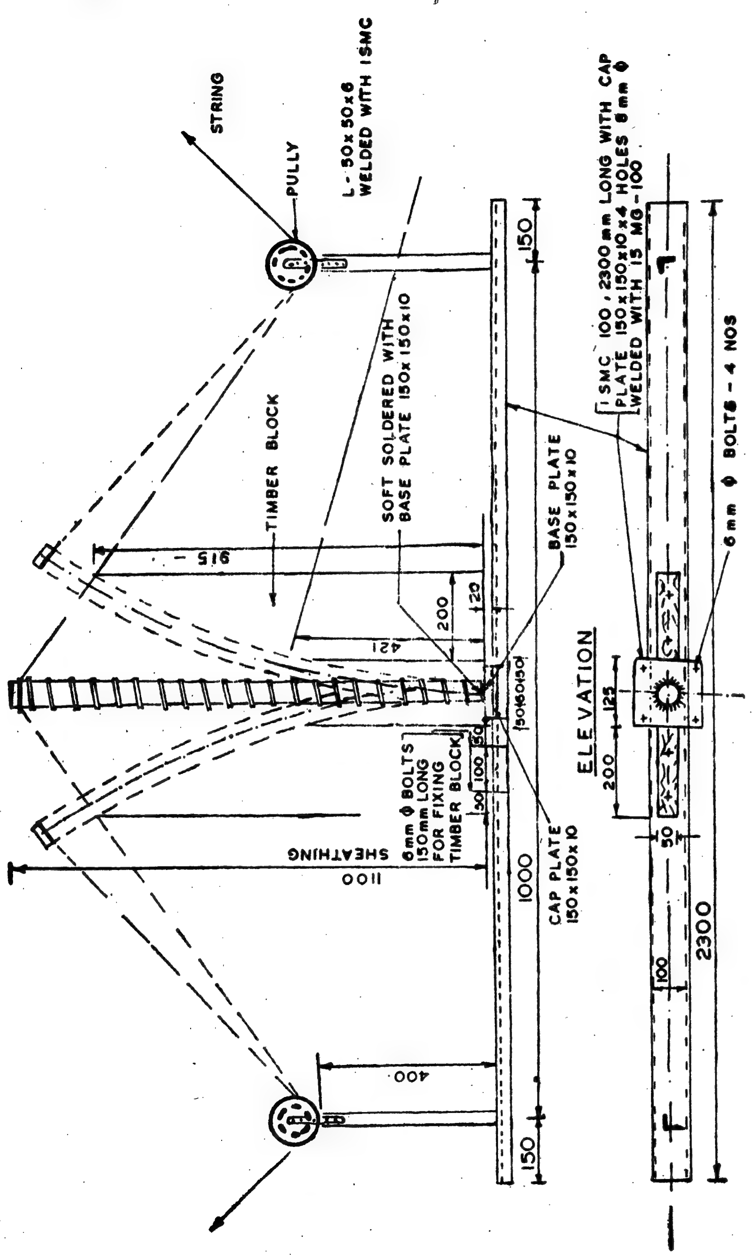


Fig. 5. Workability test

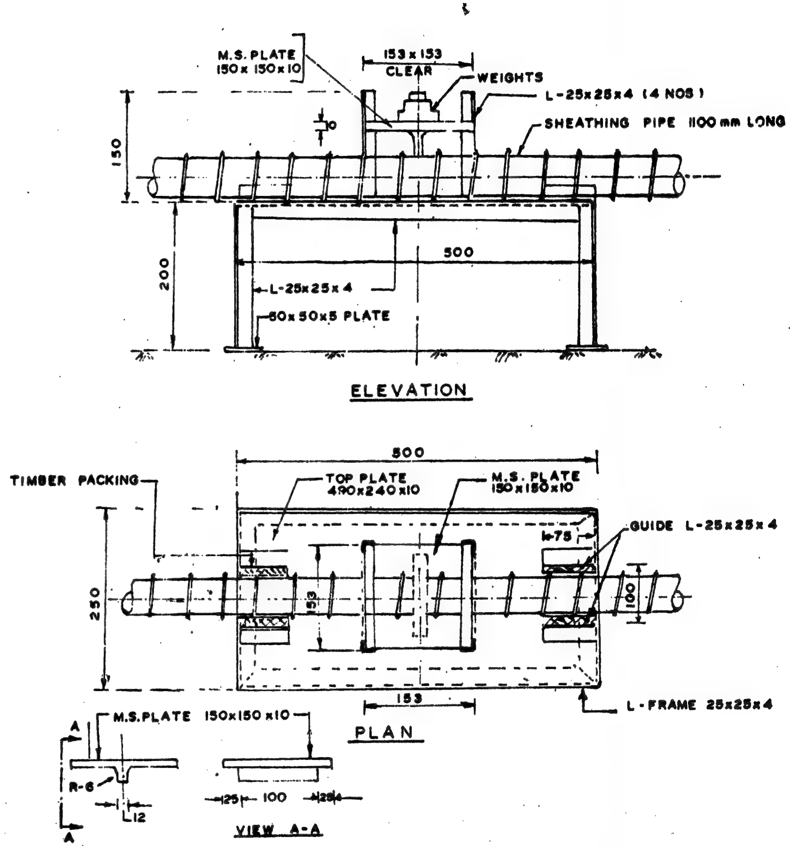


Fig. 6. Transverse load rating test

6. TENSION LOAD TEST

The test specimen is subjected to a tensile load. The hollow core is filled with a wooden circular piece having a diameter of 95 per cent of the inner dia of the sample to ensure circular profile during test loading, Fig. 7.

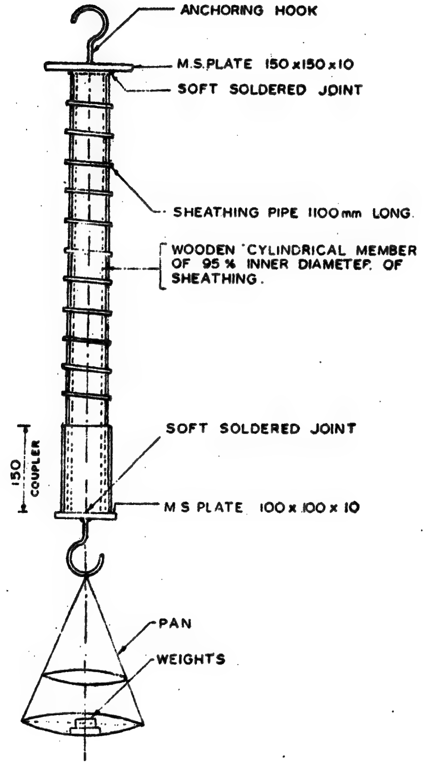


Fig. 7. Tension load test

A coupler is screwed on and the sample loaded in increments, till specified load. If no deformation of the joints nor slippage of couplers is noticed, the test shall be considered satisfactory :

Dia in mm	*Load*
25 to 35	300 N
35 to 45	500 N
45 to 55	800 N
55 to 65	1100 N
65 to 75	1400 N
75 to 85	1600 N
85 to 90	1800 N

7. WATER LOSS TEST

The sample is sealed at one end. The sample is filled with water and after sealing, the end is connected to a system capable of applying a pressure of 0.05 MPa, Fig. 8 and kept constant for 5 minutes, hand pump and pressure gauge or stand pipe system can be used.

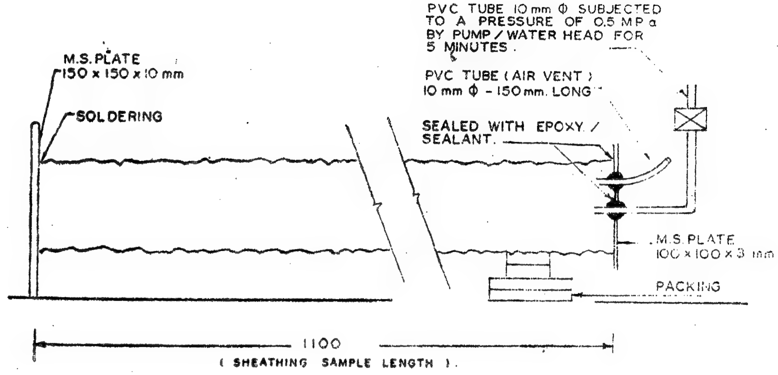


Fig. 8. Test for water loss study

The sample is acceptable if the water loss does not exceed 1.5 per cent of the volume. The volume is worked out as follows :

Another sample 500 mm long is sealed at one end and the volume of hollow space arrived at by pouring water from a measuring cylinder.

The computation of relative profile volume is worked out as follows :

V_a — Premeasured quantity of water in a measuring cylinder

V_b — Balance quantity of water left in the cylinder after completely filling of the test sample

Actual Volume 'V_p' $= V_a - V_b$

$$\text{Relative Profile Volume} = \frac{V_p - \dfrac{\pi\phi^2 l}{4}}{\pi\phi l}\ \text{cm}^3/\text{cm}^2$$

where l is length of specimen and ϕ internal nominal dia. of sheathing.

Appendix 2

RECOMMENDED PRACTICE FOR GROUTING OF POST TENSIONED CABLES IN PRESTRESSED CONCRETE BRIDGES

1. GENERAL

1.1. The recommendations cover the cement grouting of post-tensioned tendons of prestressed concrete members of bridges. This also covers some of the essential protective measures to be adopted for minimising corrosion in PSC bridges.

1.2. The purpose of grouting is to provide permanent protection to the post-tensioned steel against corrosion and to develop bond between the prestressing steel and the surrounding structural concrete. The grout ensures encasement of steel in an alkaline environment for corrosion protection and by filling the duct space, it prevents water collection and freezing.

2. MATERIALS

2.1. **Water**

Only clean potable water free from impurities conforming to Clause 3.3. of this criteria shall be permitted. No sea or creek water is to be used at all.

2.2. **Cement**

Ordinary Portland Cement should be used for preparation of the grout. It should be as fresh as possible and free from any lumps. Pozzolana cement shall not be used.

2.3. **Sand**

It is not recommended to use sand for grouting of prestressing tendons. In case the internal diameter of the ducts exceed 150 mm, use of sand may be considered. Sand, if used, shall conform to IS: 383 and shall pass through IS Sieve No. 150. The weight of sand in the grout shall not be more than 10 per cent of the weight of cement, unless proper workability can be ensured by addition of suitable plasticizers.

2.4. **Admixtures**

Acceptable admixtures conforming to IS: 9102 may be used if tests have shown that their use improves the properties of grout, i.e. increasing fluidity, reducing bleeding, entraining air or expanding the grout. Admixtures must not contain chlorides, nitrates, sulphides, sulphites or any other products which are likely to damage the steel or grout. When an expanding agent is used, the total unrestrained expansion should not exceed 10 per cent. Aluminium powder as an expanding agent is not recommended for grouting because its long term effects are not free from doubt.

2.5. **Sheathing**

2.5.1. For specifications of sheathing, Clause 3.6. of this criteria may be referred to.

2.5.2. **Grout openings or vents**

(a) All ducts should have grout openings at both ends. For this purpose special openings should be provided where such openings are not available at end anchorages. For draped (curved) cables crown points should have a grout vent. For draped cables longer than 50 m grout vents or drain holes may be provided at or near the lowest points. It is a good practice to provide additional air vents at suitable intervals. All grout openings or vents should include provisions for preventing grout leakage.

(b) Standard details of fixing couplers, inlets, outlets and air vents to the duct/anchorage shall be followed as recommended by the supplier of the system of prestressing.

2.5.3. Ducts should be securely fastened at close intervals. All unintended holes or openings in the duct must be repaired prior to concrete placing. The joints of the couplers and the sheathing should be made water proof by use of tape or similar suitable system capable of giving leak proof joints. Grout openings and vents must be securely anchored to the duct and to either the forms or to reinforcing steel to prevent displacement during concreting operations due to weight, buoyancy and vibrations

2.5.4. Ducts require very careful handling as, being of thin metal, they are susceptible to leakage due to corrosion in transit or storage, by tearing ripping in handling particularly when placed adjoining to reinforcing steel, by pulling apart of joints while inserting tendons prior to concreting, or by accidental puncturing while drilling for form ties/inserts. Ducts are also liable to damage by rough use of internal vibrator and sparks from welding being done close by.

3. EQUIPMENT

3.1. **Grout agitator**

It is essential that the grout is maintained in a homogenous state and of uniform consistency so that there is no separation of cement. It is, therefore, necessary that the grout be continuously agitated by a suitable mixer with a minimum speed of 1000 RPM and travel of discharge not exceeding 15 m per second.

3.2. **Grout pump**

The pump should be a positive displacement type and should be capable of injecting the grout in a continuous operation and not by way of pulses. The grout pump must be fitted with a pressure gauge to enable pressure of injection to be controlled. The minimum pressure at which grout should be pumped shall be 0.3 MPa and the grout pump must have a relief arrangement for bypass of the grout in case of build up of prsssure beyond 1 MPa. The capacity of the grout pump should be such as to achieve a forward speed of grout of around 5 to 10 metres per minute. The slower rates are preferable as they reduce the possibility of occurrence of voids. If the capacity of the pump is large, it is usual to grout two or more cables simultaneously through a common manifold.

Use of hand pumps for grouting is not recommended. Use of compressed air operated equipment for injection is prohibited as it is likely that there will be some air entrapped in grout.

3.3. **Water pump**

Before commencement of grouting, a stand by direct feed high pressure water pump should be available at site for an emergency. In case of any problem in grouting the ducts, such pump shall immediately be connected to the duct and all grout flushed by use of high pressure water flushing. It is, therefore, necessary to have adequate storage of clean potable water for operation of the water pump for such emergencies.

3.4. **Grout screen**

The grouting equipment should contain a screen having a mesh size of IS: 106 (IS:150 if sand is used). Prior to introduction into the grout pump, the grout should be passed through such screen. This screen should be easily accessible for inspection and cleaning.

3.5. **Connections and air vents**

Standard details of fixing inlets, outlets, and air vents to the sheathing and/or anchorage should be followed as recommended by specialist supplier of the system of prestressing. In general, all connections are to be of the "Quick couple" type and at change of diameters suitable reducers are to be provided.

4. PROPERTIES OF THE GROUT

4.1. Water/cement ratio should be as low as possible, consistent with workability. This ratio should not normally exceed 0.45.

4.2. Before grouting, the properties of the grout mix should be tested in a laboratory depending on the facilities available. Tests should be conducted for each job periodically. The recommended test is described below.

4.2.1 **Compressive strength :** The compressive strength of 100 mm cubes of the grout shall not be less than 17 MPa at 7 days. Cubes shall be cured in a moist atmosphere for the first 24 hours and subsequently in water. These tests shall be conducted in advance to ascertain the suitability of the grout mix.

5. MIXING OF GROUT

5.1. Proportions of materials should be based on field trials made on the grout before commencement of grouting, but subject to the limits specified above. The materials should be measured by weight.

5.2. Water should be added to the mixer first, followed by Portland cement and sand, if used. Admixture if any, may be added as recommended by the manufacturer.

5.3. Mixing time depends upon the type of the mixer but will normally be between 2 and 3 minutes. However, mixing should be for such a duration as to obtain uniform and thoroughly blended grout, without excessive temperature increase or loss of expansive properties of the admixtures. The grout should be continuously agitated until it is injected.

5.4. Once mixed, no water shall be added to the grout to increase its fluidity.

5.5. Hand mixing is not permitted.

6. GROUTING OPERATIONS

6.1. **General**

(a) Grouting shall be carried out as early as possible but not later than 2 weeks of stressing a tendon. Whenever this stipulation cannot be complied with for unavoidable reasons, adequate temporary protection of the steel against corrosion by methods or products which will not impair the ultimate adherence of the injected grout should be ensured till grouting. The sealing of the anchorage ends after concreting is considered to be a good practice to prevent ingress of water. For structures in aggressive environment, sealing of the anchorage ends is mandatory.

Notes : 1. Application of some patented water soluble oils for coating of steel/VPI powder injection/sending in of hot, dry, oil-free compressed air through the vents at frequent intervals have shown some good results.

2. Some of the methods recommended for sealing of anchorages are to seal the openings with bitumen impregnated gunny bag or water proof paper or by building a brick pedestal plastered on all faces enclosing the exposed wires outside the anchorages.

(b) Any traces of oil if applied to steel for preventing corrosion should be removed before grouting operation.

(c) Ducts shall be flushed with water for cleaning as well as for wetting the surfaces of the duct walls. Water used for flushing should be of same quality as used for grouting. It may, however, contain about 1 per cent of slaked lime or quick lime. All water should be drained through the lowest drain pipe or by blowing compressed air through the duct.

(d) The water in the duct should be blown out with oil free compressed air.

Blowing out water from duct for cables longer than 50 m draped up at both ends by compressed air is not effective, outlet/vent provided at or near the lowest point shall be used to drain out water from duct.

(e) The connection between the nozzle of the injection pipe and duct should be such that air cannot be sucked in.

(f) All outlet points including vent openings should be kept open prior to commencement of injection grout.

(g) Before grouting, all air in the pump and hose should be expelled. The suction circuit of the pump should be air-tight.

6.2. **Injection of grout**

(a) After mixing, the grout should be kept in continuous movement.

(b) Injection of grout must be continuous and should not be interrupted.

(c) For vertcial cable or cables inclined more than 60° to the horizontal injection should be effected from the lowest anchorage or vent of the duct.

(d) The method of injection should ensure complete filling of the ducts. To verify this, it is advisable to compare the volume of the space to be filled by the injected grout with the quantity of grout actually injected.

(e) Grouting should be commenced initially with a low pressure of injection of upto 0.3 MPa increasing it until the grout comes out at the other end. The grout should be allowed to flow freely from the other end until the consistency of the grout at this end is the same as that of the grout at the injection end. When the grout flows at the other end, it should be closed off and build up of pressure commenced. Full injection prossure at about 0.5 MPa shall be maintained for at least one minute before closing the injection pipe. It is a recommended practice to provide a stand pipe at the highest point of the tendon profile to hold all water displaced by sedimentation or bleeding. If there is a build up of pressure much in excess of 1 MPa without flow of grout coming at the other end, the grouting operation should be discontinued and the entire duct flushed with high pressure water. Also, the bypass system indicated in para 3.2. above is essential for further safety.

(f) In the case of cables draped downwards e.g. in cantilever construction simultaneous injection from both ends may be adopted Fig. 9.

(g) Grout not used within 30 minutes of mixing should be rejected.

(h) Disconnection is facilitated if a short length of flexible tube connects the duct and injection pipe. This can be squeezed and cut off after the grout has hardened.

7. PRECAUTIONS AND RECOMMENDATIONS FOR EFFECTIVE GROUTING

(a) In cold and frosty weather, injection should be postponed, unless special precautions are taken. If frost is likely to occur within 48 hours after injection, heat must be applied to the member and maintained for at least 48 hours after injection so that the temperature of the grout does not fall below 5°C. Prior to commencement of grout, care must be taken to ensure that the duct is completely free of frost /ice by flushing with warm water, but not with steam.

(b) When the ambient temperature during the day is likely to exceed 40°C, grouting should be done in the early morning or late evening hours.

(c) When the cables are threaded after concreting, the duct must be temporarily protected during concreting by inserting a stiff rod or a rigid PVC pipe or any other suitable method.

(d) During concreting, care shall be taken to ensure that the sheathing is not damaged. Needle vibrators shall be used with extreme care by well experienced staff only, to ensure the above requirements.

(e) It is a good practice to move the cables in both directions during the concreting operations. This can easily be done by light hammering the ends of the wires/strands during concreting. It is also advisable that 3 to 4 hours after concreting the cable should be moved both ways through a distance of about 20 cms. With such movement, any leakage of mortar which has taken place in spite of all precautions, loses bond with the cables, thus reducing the chance of blockages. This operation can also be done by fixing prestressing jacks at one end pulling the entire cable and then repeating the operation by fixing the jack at the other end.

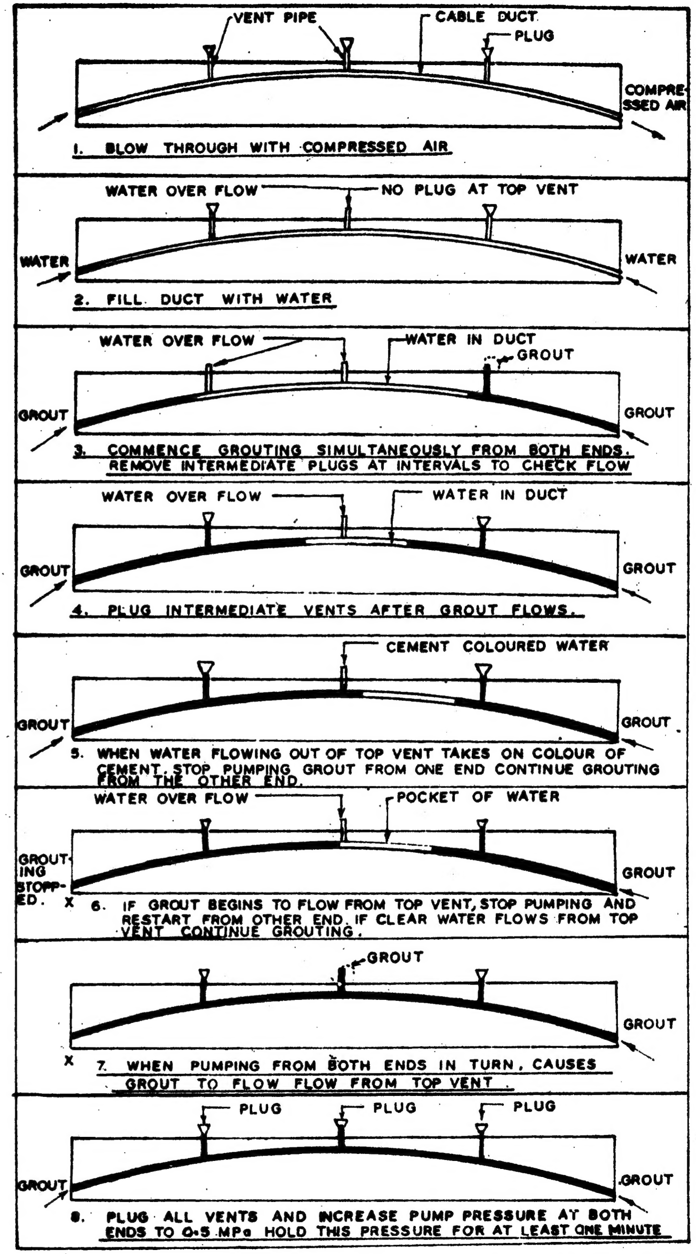


Fig. 9. Procedure for grouting of cables draped downwards

(f) The cables to be grouted should be separated by as much distance as possible.

(g) In case of stage prestressing, cables tensioned in the first stage should not remain ungrouted till all cables are stressed. It is a good practice, while grouting any duct in stage prestressing, to keep all the remaining ducts filled up with water containing 1 per cent lime or by running water through such ducts till the grout has set. After grouting the particular cable, the water in the other cables should be drained and removed with compressed air to prevent corrosion.

(h) Care should be taken to avoid leaks from one duct to another at joints of precast members.

(i) End faces where anchorages are located are vulnerable points of entry of water. They have to be necessarily protected with an effective barrier. Recesses should be packed with mortar concrete and should preferably be painted with water proof paint.

(j) After grouting is completed, the projecting portion of the vents should be cut off and the face protected to prevent corrosion.

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